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Methods for life extension of multi-storey car park buildings

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Abstract

Multi-storey car park buildings with steel–concrete composite floor structures are widespread in construction due to the distinct benefits of the implemented structural system. Although there are various possibilities for accomplishing composite action between a steel beam and concrete slab, some solutions as friction-grip bolts commonly applied in the 1970s, did not show good durability. An example is the multi-storey car park building "Obilićev venac" in Belgrade. The case study presents two methods for life extension of the car park building structure. Both approaches focus on applying welded headed studs with the primary intention to provide a durable shear connection. While the first method has been applied during the structure reconstruction, the second method is proposed as the novel solution for implementation in steel–concrete composite floors. The application of the proposed connection is discussed and results of the conducted experimental and numerical research are presented.

Keywords: steel–concrete composite structures, demountable shear connections, friction-grip bolts, welded headed studs, reconstruction, profiled steel sheeting, durability

Introduction

Steel–concrete composite structures are a popular option for the design of multi-storey car park buildings. Besides fast construction, steel beams satisfy requests considering long spans enabling column-free space for vehicle movement and parking. These structures are economical and lightweight, and therefore convenient for a foundation. Moreover, steel–concrete composite buildings may be demountable, enabling the reuse of certain structural elements or the whole structure on another location and simplifying the structure rehabilitation and repair. The positive impact of demountable steel–concrete systems on reducing emissions and saving resources has been proved [1].

Precast concrete slabs are a widespread solution for application in composite steel–concrete floors due to the reduction of construction time. Several car park buildings with precast solid concrete slabs connected to steel beams via friction-grip bolts were built in Serbia in the 1970s. These floor structures were characterised by easy mounting and fast execution, avoiding concrete casting on the construction site. In addition, the applied solution enabled structure demounting and relocation. However, over a few decades, the serviceability of these buildings got seriously affected, which could be attributed to the durability problems of the specific shear connecting system that had been applied.

In the paper, this issue is discussed in the example of the car park building "Obilićev venac" in Belgrade, Serbia (Figure 1). The original structural design and the state at the time before reconstruction are described. Afterwards, two methods for life extension of the multi-storey car park building are presented. The first method has been released through the building reconstruction project, with the intention to keep all structural elements that were not heavily damaged and accomplish satisfying durability of the structure. However, the developed shear connection with welded headed studs is not demountable, preventing the reuse of the whole structure on another location. Therefore, the second method is presented as the alternative solution for developing demountable shear connections, and it is compared with the original and the reconstruction design, explaining its benefits. The proposed solution of the shear connection consists of welded headed studs and bolts. Possible application of the demountable connection in the car park building is discussed, and the connection shear behaviour is presented through results of experimental and numerical analyses.

Car park building "Obilićev venac": The original design

The car park building "Obilićev venac" was built in 1975. During the construction, the designer and supervising company were Krupp Industrie- und Stahlbau. Initially, the building had five storeys and a total area of 15 000 m^2. The main bearing structure consisted of steel frames, according to the layout given in Figure 2. The main beams of the cross-section IPE 400 and IPE 360, spaced on 2.3 m, were designed to span 15 m and 10 m, respectively.

According to the design, the floor structure was constructed from demountable precast solid concrete slabs and steel beams, that were connected and acted compositely. Concrete slabs were of a depth of 100 mm, dimensions 2.3 x 5.0 m. For connecting slabs and steel beams, the Krupp-Montex system consisted of high strength friction-grip bolts M16, inserted at a distance of 300 mm, was applied. The connection layout according to the original design is presented in Figure 3. Bolts were placed in PVC tubes of a diameter of 25 mm, chosen to satisfy necessary execution tolerances. Helical reinforcement around tubes was applied to prevent concrete damage due to high local stresses induced by bolt preloading. No injection was used.

In 2016, nearly 40 years after the construction, the investor requested the repair of the existing structure and the addition of two storeys. At that moment, the serviceability of the car park building was affected by significant deformations of the floor structure. The maximum measured value of the vertical deflection in the beam mid-span was more than 80 mm (over L/187). Monitoring released between 2010 and 2012 showed the increase of deflections up to 10 mm in the most critical beams, with the tendency of progressive deflection growth. Large deflections resulted from the disappearance of the composite action between steel beams and concrete slabs induced by the failure of shear connectors. The visual assessment showed that friction-grip bolts had slipped in holes and PVC tubes after the slip resistance had been exceeded, which had been influenced by the fading of the coating layer on the contact between a steel flange and a concrete slab. Furthermore, a very high level of corrosion of bolts was noticed. The comparison of the original bolt M16 inserted during construction and the new bolt M16, given in Figure 4, shows a significant reduction in the cross-sectional area. Bolt corrosion was mainly induced by water accumulated in PVC tubes as no floor protection had been applied on the slab top surface.

Method 1: The released reconstruction

The reconstruction of the car park building included several different construction works: strengthening of the existing bearing structure, rehabilitation and reconstruction of the floor structure, replacement of the damaged precast concrete slabs and application of fire and corrosion protection on the steel structure. In addition, all installations were replaced and new facade was installed.

Most of the concrete slabs were not heavily damaged, and their complete replacement was not needed. Therefore, the following repair method was adopted for the reconstruction of the floor structure: the existing bolts were supposed to be left in slabs or, if damaged, replaced by new ones, while welded headed studs of a diameter of 22 mm and height of 90 mm were designed as new shear connectors. Welded headed studs were inserted at the distance of 300 mm, in between bolts. The layout of the developed connection is presented in Figure 5. Despite the lack of bearing function in shear connection, bolts were left in slabs to enable the transversal connection between steel beams and concrete slabs and accomplish the in-plane stiffness of the floor structure. The same solution with welded headed studs as shear connectors was applied to the heavily damaged concrete slabs after their replacement.

At first, the suggested reconstruction method was conducted on the beams on level zero, followed by experimental testing of the static and dynamic response of the floor structure. Measurements confirmed the designed behaviour of repaired composite floor beams. Therefore, the proposed method was accepted and applied in the entire building. The reconstruction was released from the upper storeys. The support structure, which consisted of proppings and hydraulic presses of a capacity of 10 t, was installed. Steel beams were supported in two points along their span, as shown in Figure 6. Firstly, old bolts were unscrewed to annulate any potential composite action in beams. Secondly, girders were cambered up to 30 mm to compensate for the existing deflections. Concrete cores of a diameter of 100 mm were extracted at a distance of 300 mm. Headed studs were welded to the top flange of steel beams, according to Figure 7, followed by grouting of holes with expansive mortar. In the end, bolts were tightened, applying 50% of the maximum pretension force. Epoxy floor coating was installed to accomplish satisfying durability of the floor structure.

The method for repair of steel–concrete composite beams, including welded headed studs installed in pre-drilled holes in the concrete slab, was accepted as the work- and cost-effective solution that would enable the appropriate durability of the structure. The adopted method is characterised by relatively fast execution and low requests of resources considering material and construction work. The method including total replacement of bolts with new ones that would have a shear connecting role was rejected. One of the reasons lies in the local damage of concrete slabs around friction-grip bolts, requiring further rehabilitation work and slab replacement to enable the proper realisation of shear forces.

However, the applied reconstruction method transformed the original demountable steel–concrete composite system to a non-demountable one, disabling reuse and relocation of the complete structure. Hence, before implementing the proposed method, the planned life cycles of a building should be carefully considered. In the particular case of the car park building "Obilićev venac", the building demolition and relocation were unlikely to be requested in future according to the analysed traffic demands in Belgrade. Therefore, demountability of the structure was not the client request for the reconstruction.

Method 2: The alternative solution

To develop demountable steel–concrete composite floors, many technical solutions have been proposed suggesting the application of various bolted shear connectors instead of headed studs [2]. Widely investigated friction-grip bolts [3–7] are preloaded and transfer shear force through friction between a concrete slab and a steel beam. But, as observed in the example of this car park building, the persistence of pretension force in friction-grip bolts over time could be questionable. As a result, sudden slip occurs, followed by additional deformations of the girder. Alternatively, the application of demountable threaded headed studs [8–10] or bolts that are not preloaded was suggested. These connections are characterised by lower stiffness and the initial slip induced by bolt-to-hole clearances. The solution with bolts with nuts embedded in concrete [11] increases the stiffness of the shear connection, but the problem of the initial slip, inducing deflections of the girder in the construction stage, is still present. Similar behaviour was observed for connectors with a coupler and two bolts, one placed in the concrete slab and another passing through a beam flange hole [6,12]. Injection of the resin in holes of the steel flange could improve the performance of bolts and prevent initial slip due to bolt-to-hole clearances, as shown for the coupler system [6]. Another approach for solving this problem was suggested by locking-nut and friction-based shear connectors [13,14] installed in special tools with conical nuts placed in conical holes of a beam flange. Furthermore, blind bolts were investigated for application in demountable floors [15] and for strengthening steel–concrete composite beams [16,17]. Also, some recent research was focused on innovative bolted connectors with clamps [18].

In addition to the mentioned solutions, a demountable connection with welded headed studs and bolts is proposed. The solution consists of a composite concrete slab cast in open trough profiled steel sheeting, a pair of steel angles and a steel beam. As well as in the original floor structure with friction-grip bolts, slabs are discontinuous over the beam. The connection layout is presented in Figure 8. The composite concrete slab is connected to the angle via welded headed studs, and the angle is connected to the upper flange of a steel beam via bolts. Therefore, composite action is released indirectly from the steel beam to the concrete slab, along two shear planes: "steel beam–angle" and "angle–concrete slab".

For implementation in the car park building, the proposed solution requires the replacement of solid concrete slabs with new composite slabs with profiled steel sheeting. Angles should be placed over beams, and bolts should be installed. Headed studs may be welded to angles through profiled sheeting, or sheeting with pre-punched holes may be used. Afterwards, the concrete slab may be cast in situ. To dismantle the structure, bolts need to be untightened, and the slab becomes separated from the steel beam, according to Figure 9. In the second life cycle of the building, the complete floor structure, including composite slabs and steel beams, can be reused.

The material consumption in the case of the applied solutions before and after reconstruction and the alternative proposals is summarised in Table 1. Steel grade, concrete class and profile of a steel cross-section are presented according to the original project. Proposed solutions are provided considering three different types of an open trough profiled metal decking commercially available in Europe, marked as Type 1 [19], Type 2 [20] and Type 3 [21]. All three selected types have an overall depth of approximately 60 mm, which is considered an optimum solution for the analysed composite slabs spanning 2.3 m. The overall slab depth is set to 120 mm. General advantages of profiled steel sheeting are well-known, including high rigidity and low weight, reflected in the convenient manipulation, high adaptability and flexibility and dual functionality: a safe working platform and a bearing element during service life [22]. The application of composite slabs leads to savings in concrete consumption and reduction of the overall structural weight.

Design resistance of headed studs used with different profiled steel sheeting, calculated according to EN 1994-1-1 [23], and design shear resistance of bolts, calculated according to EN 1993-1-8 [24], are given and compared in Table 2. The initial requirement is to keep the bolts' resistance higher than headed studs' in order to avoid large bolt deformations and failure and provide a connection response close to the response of non-demountable connection with welded headed studs. In other words, it is presumed that bolts' response is elastic at ultimate loads, while headed studs' deformation is in the plastic domain. Therefore, the proposed demountable shear connection contains headed studs of a diameter of 16 mm and height of 100 mm, and bolts M16 or M20 of the grade 8.8. A pair of headed studs is set in each sheeting rib, whereas a pair of bolts is set in-between every second rib. For the beams spanning 15 m and 10 m, the total number of headed studs is in the range of 90–146 and 60–96, respectively, depending on the selected metal decking. The total number of bolts is approximately two times smaller. Bolt prediction on the longitudinal spacing twice longer than the spacing between headed studs leads to savings in execution time and costs.

Although the proposed floor design has a larger number of connectors compared with the applied solutions before and after the car park building reconstruction, the benefit of the proposed method is the absence of any special tools and additional parts as specially designed steel plates, PVC tubes and helical reinforcement, which are needed for shear connections with friction-grip bolts. Both types of connectors in the suggested demountable shear connection, headed studs and bolts, are commercially available standard products commonly used in construction.

Before implementation, special attention should be paid to connection detailing with the accent on the connection between the composite concrete slab and angles, accomplished by headed studs. Parameters as angle thickness and stud-to-edge distance should be carefully selected to provide the appropriate connection response. The angle thickness is set to 8 mm to satisfy EN 1994-1-1:2004 [23] requirement that the thickness of the component to which the stud is welded should be at least 0.4*d*, where *d* is the headed stud shank diameter. Bearing resistance at bolt holes for the angle thickness of 8 mm is greater than bolt shear resistance, and it is not considered critical. The distance between the headed stud and slab edge in the transverse direction is set to 4*d* to keep the connector above the steel flange. U-bars of a diameter of 8 mm are added around headed studs to prevent concrete splitting failure and reinforce the concrete nearby the slab edge. The validity of the adopted detailing is experimentally tested.

Load–slip behaviour

To examine the load–slip performance of the proposed demountable connection, push-out tests under static loading were performed according to EN 1994-1-1, Annex B [23]. Specimens were developed applying the profiled steel sheeting Type 1 [19] and following the proposed design (series DLU). Moreover, the corresponding non-demountable connection with welded headed studs was tested as a control series (series S). Specimens' layout is given in Figure 10. Non-demountable specimens were formed of two concrete slabs connected to the steel profile HEB 260 by welded headed studs. Demountable specimens consisted of four concrete slabs connected to HEB 260 via angles using stud and bolt connectors.

The suggested demountable connection design was followed during specimen preparation – headed studs of a diameter of 16 mm and height of 100 mm were applied, and the previously described detailing was implemented. The exception was made in the case of bolt diameter. Bolts were installed in front of the first row and behind the second row of headed studs to accomplish shear force transfer from the steel profile to concrete slabs. Therefore, the total number of bolts in the specimen was equal to the number of headed studs. The bolt diameter was reduced from 16 mm to 12 mm to accomplish a similar ratio in the total resistance of bolt and stud connectors as planned for implementation in the car park building.

Firstly, specimens were subjected to 25 loading cycles in the range from 5% to 40% of the expected failure load. Afterwards, loading was applied in one step, controlling that failure does not appear in less than 15 minutes. The force, vertical slip and horizontal separation were monitored during testing with the frequency of 1 Hz. Force was measured by a load cell. Displacements were measured using LVDT sensors, according to Figure 10: four sensors were used to measure the horizontal separation between the concrete slab and steel profile, and four sensors were used to measure the vertical slip between the steel profile and concrete slabs. In demountable specimens, additional four sensors were used to measure the vertical slip between angles and the slab.

All specimens failed due to concrete damage, reflected in the pull-out failure, as presented in Figure 11. Headed stud connectors had a certain deformation. After the testing, bolts were easily demounted, and no deformation was observed. Angles were unaffected, as well.

The averaged load–sip curves obtained during push-out testing of two demountable and two non-demountable specimens are presented in Figure 11. For the demountable connection, two load–slip curves are plotted, as the vertical slip was measured between the steel profile and slab and between angles and the slab. A summary of the measured ultimate loads P_{ult}, stiffness corresponding to $0.7P_{\mathrm{ult}}$ and slip capacity is given in Table 3.

Comparing the results, a very good match in the ultimate loads of demountable and non-demountable specimens is observed, implying that the resistance of the proposed connection with a discontinuous slab over the beam agrees with the resistance of the connection with a continuous slab over the support. The key differences in the response of demountable and non-demountable connections are reflected in the slip and stiffness. The demountable connection has a larger slip capacity of more than 6 mm. Furthermore, demountable specimens have approximately four times smaller stiffness than non-demountable ones as a result of the bolt slip inside the holes. In the case of the tested specimens with a bolt hole diameter of 13 mm and a bolt diameter of 12 mm, the initial bolt slip is approximately 0.5 mm (Figure 11). After the initial clearances in holes are overcome, slip in the shear plane "angle–slab" starts increasing. The total slip of the demountable specimen is a sum of the slip in the plane "angle–slab" and "beam–angle". Slip in the plane "beam–angle" is dominant in the initial loading stage, whereas the plane "angle–slab" gets activated for higher load levels. The load–slip response of the non-demountable connection and load–slip curve for shear in the plane "angle–slab" of the demountable connection match well in the pre-ultimate domain. Therefore, it may be confirmed that headed stud connectors welded to angles and profile flange act similarly.

Numerical analysis was conducted to investigate deformations and stress distribution in different connection parts at serviceability and ultimate loads. Models of push-out specimens were developed in the commercial finite element software [25], as presented in Figure 12. The double symmetry boundary conditions were set to save computation time. Material models were defined according to the measured material properties: concrete (for model S: $f_{\mathrm{c,cube}}$ = 43.7 MPa; for model DLU: $f_{\mathrm{c,cube}}$ = 46.6 MPa), steel profile (f_y = 297 MPa, f_u = 419 MPa), angles (f_y = 357 MPa, f_u = 521 MPa), headed studs (f_y = 421 MPa, f_u = 509 MPa), bolts (f_y = 929 MPa, f_u = 967 MPa), profiled sheeting (f_y = 348 MPa, f_u = 408 MPa). As the governing failure mode was a concrete pull-out failure, numerical simulation results are dominantly affected by the concrete material model. The behaviour of steel parts was modelled according to the experimental stress–strain curves, defining material response in the elastic and plastic domain. To simulate concrete behaviour adequately, the concrete damage plasticity model was applied according to [11]. The concrete stress–strain curve up to the strain of ε_{cu1} = 3.5‰ was defined according to EN 1992-1-1 [26]. For the strain ε_c > 3.5‰, the stress–strain curve for concrete was described according to [11], using the coefficients α = 8, α_{tD} = 0.5, α_{tE} = 0.6, and adopting the values of strains ε_{cuE} = 0.05, ε_{cuF} = 0.20. General contact was used to model contacts between different parts, setting the "hard" contact in the normal direction and penalty contact in the tangential direction with appropriate friction coefficients (0.20–0.45). Embedded constrain was applied to reinforcement. Three different finite elements were used: C3D8R for solid parts, S4R for profiled steel sheeting modelled as a shell part, and T3D2 for reinforcement bars modelled as beam parts. The smallest size of elements was 2 mm applied in the area around headed studs, while the largest elements of 10 mm were used in periphery regions of the model.

Finite element models were validated against experimental results. Comparisons between ultimate loads and load–slip curves are given in Table 4 and Figure 13. A good match in the response of experimental specimens and numerical models was accomplished.

Propagation of concrete cracks presented in Figure 14a indicates concrete pull-out failure and separation of the concrete cone from the rest of the slab. The same failure mechanism is observed in the model of the non-demountable connection. Deformation of bolt connectors remains minor at the slip of 6 mm, while deformation of headed studs is more pronounced, according to Figure 14b. Stresses in bolts are below the yield strength. However, the maximum stress in headed studs is above the yield strength. The concentration of stresses is observed at the bottom of the stud shank. Stress distribution and deformation of headed studs in the proposed demountable connection correspond to stresses and deformations in the non-demountable connection.

Adopted design, the geometry of angles and U-bars provide adequate shear resistance of the connection. Described results indicate the good performance of the proposed connection, with the potential for implementation in floor structures. However, the effects of the reduced stiffness due to bolt slip should be accounted for during beam design.

Additional deflections and bending resistance

The initial slip of bolts leads to incomplete interaction between the concrete slab and steel beam at the initial loading stage, inducing additional deflections of the steel beam in the stage before establishing composite action. This deflection may be calculated according to the equation based on the initial bolt-to-hole clearances [27] or considering the stiffness of a shear connector [28]. The average initial clearance of the adopted bolt M16 is 1 mm, assuming the bolt hole diameter of 18 mm. According to [27], it causes the additional deflections of steel beams: 18.6 mm for the beam of the profile IPE 400, spanning 15 m, and 13.6 mm for the beam of the profile IPE 360, spanning 10 m. In other words, according to deflections in the case of un-propped construction given in Table 5, for the first beam, additional deflection equals 23% of the deflection induced by dead weight in the first stage of construction, while for the second beam, it is 61% of the dead weight deflection. (For calculation, following loads have been accounted: structure self-weight, live load during construction 0.75 kN/m^2, installations 1.0 kN/m^{2,} and live load in garage 2.5 kN/m^2 [29]; all results are presented only for the profiled steel sheeting Type 1 [19], as there are no significant differences in deflections between three proposed solutions with different types of the analysed metal decking presented in Table 1.) Presented results of the dead weight slip indicate that beams spanning 15 m need to be propped during construction. When the beam is supported in two points along the span, the total mid-span deflection in the serviceability stage is 18% larger for the applied demountable connection than for the non-demountable one, calculated according to [28] and applying the experimentally obtained stiffness (Table 3).

Compared with the other shear connections with bolted connectors embedded in a concrete slab [6,30,31], the proposed demountable solution is characterised by smaller bolt-to-hole clearances and initial slip. This is influenced by smaller fabrication tolerances requested in steel structure fabrication compared with concrete casting. Therefore, the additional beam deflections for the suggested connection are smaller than in connections with bolts embedded in a concrete slab. However, these deflections should be considered during design. If necessary, deflections could be additionally reduced by bolt preloading. Alternatively, pre-cambering or propped construction could be applied as standard options for annulling deflections induced by permanent actions.

Steel angles applied in the proposed solution enable the connection demountability and act as a mould for concrete casting. Besides the mentioned, they increase bending resistance of the composite steel–concrete cross-section. Plastic resistance moments calculated for the composite cross-sections with steel profiles IPE 400, S355 and IPE 360, S235 are given in Figure 15. Resistance moments are obtained for two cases: when the resistance of steel angles of a thickness of 8 mm is neglected and when it is included in the calculation. Even though the presence of steel angles does not significantly affect the bending capacity of the composite cross-section with the steel beam IPE 400 (only 5%), for the beam IPE 360 where the plastic neutral axis lies in the concrete slab, angles increase the bending resistance even for 12%. Therefore, it is suggested to consider the contribution of angle resistance in design.

Conclusions

This paper presents two methods for life extension of multi-storey car park buildings with steel–concrete composite floor structures. As observed in the example of the car park building “Obilićev venac” in Belgrade, the shear connecting system Krupp-Montex, commonly used in car parks during the 1970s, appeared unreliable over the structure lifetime affecting the building serviceability. Two proposed methods are based on the implementation of headed studs as shear connectors in the steel–concrete composite floors. The first method includes headed studs placed in holes pre-drilled in concrete slabs. This method has been applied to the presented car park building during the structure reconstruction as the economically justified solution. However, as that method transforms the structure into non-demountable, disabling building relocation and multiple use, another method has been proposed.

The suggested demountable connection is based on the indirect transfer of a shear force, accomplished by headed studs embedded in the composite concrete slab and welded to the additional steel angle, and bolts, which connect the angle and steel flange. The proposed solution enables the reuse of the whole floor structure in the second building life cycle. Experimental push-out tests showed that the resistance of the demountable connection is equal to the resistance of the equivalent non-demountable connection with headed studs. The ductility of the proposed connection is satisfactory, while stiffness is decreased compared with the non-demountable connection. The effects of the reduced stiffness due to bolt slip should be accounted for during beam design.

Future research will be performed with the aim of developing recommendations for the design of the proposed demountable connection. Experimental and numerical studies will be extended to cover various configurations of the connection and quantify the effects of different parameters on the connection response, with the purpose to withdraw conclusions necessary for the connection implementation in the building construction.

Acknowledgements

Disclosure statement

The authors report no conflict of interest.

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Figure 1. Multi-storey car park building "Obilićev venac" after the reconstruction.

Figure 2. Plan view at levels 12 and 13.

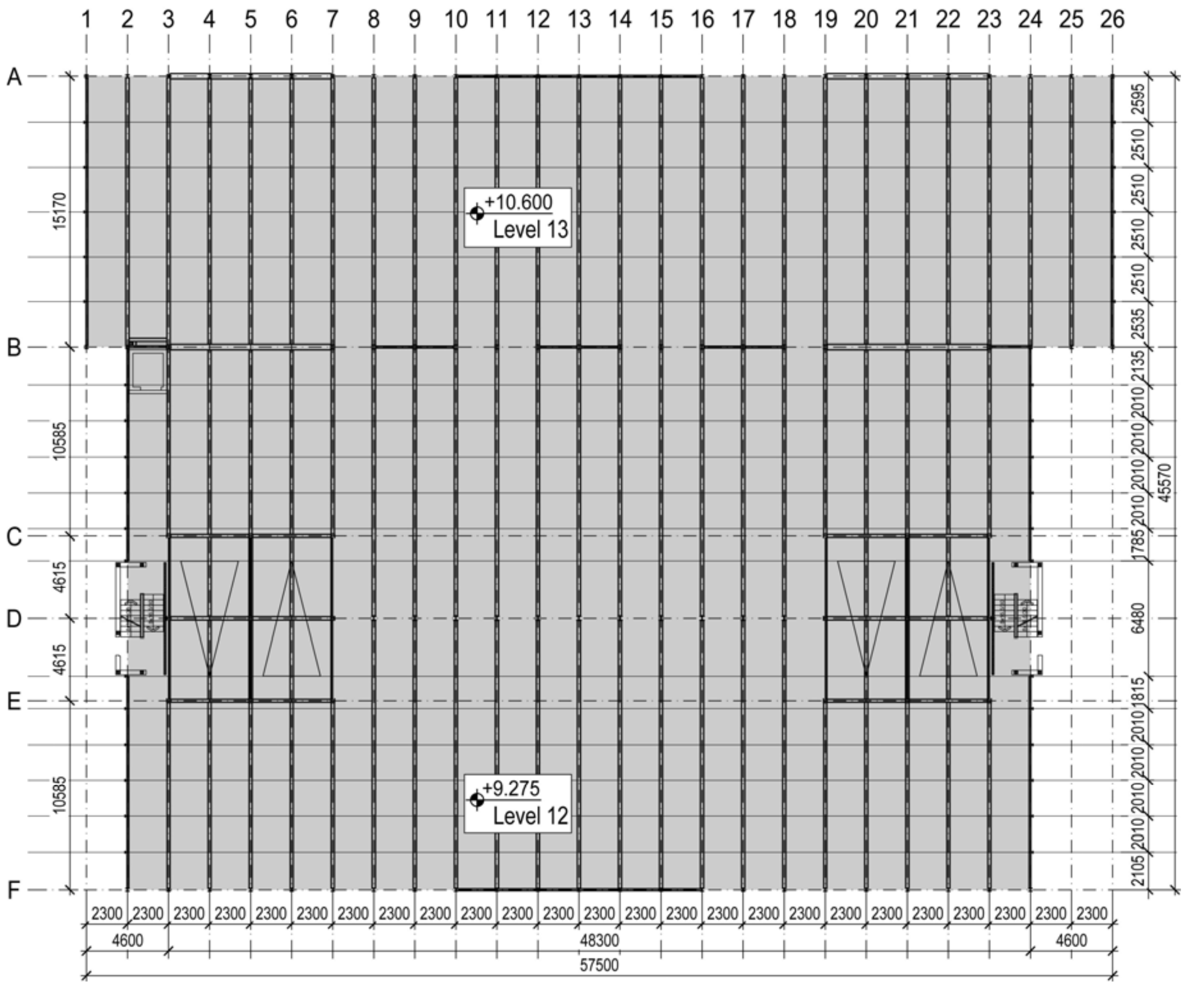

Figure 3. Connection with friction-grip bolts.

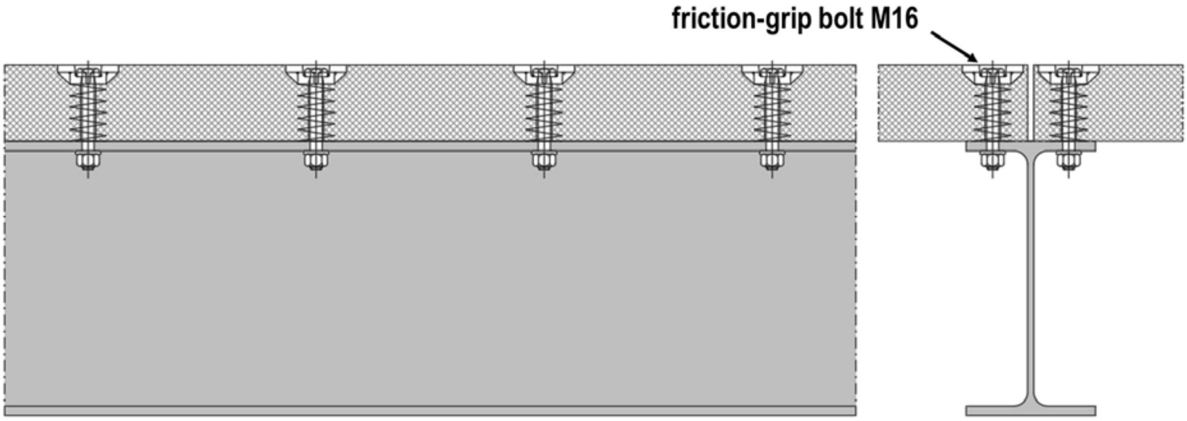

Figure 4. Comparison of the original corroded bolts with the new bolt of the same diameter.

Figure 5. Connection with welded headed studs.

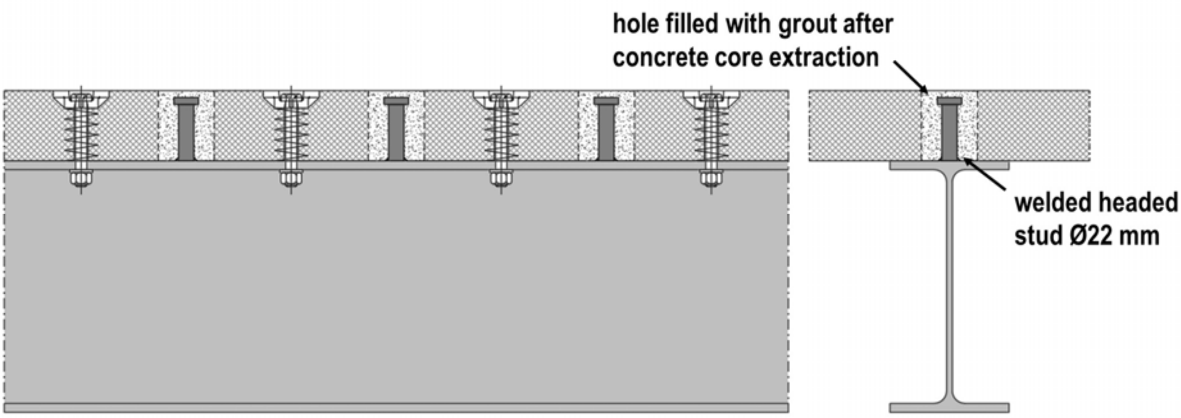

Figure 6. Car park building reconstruction.

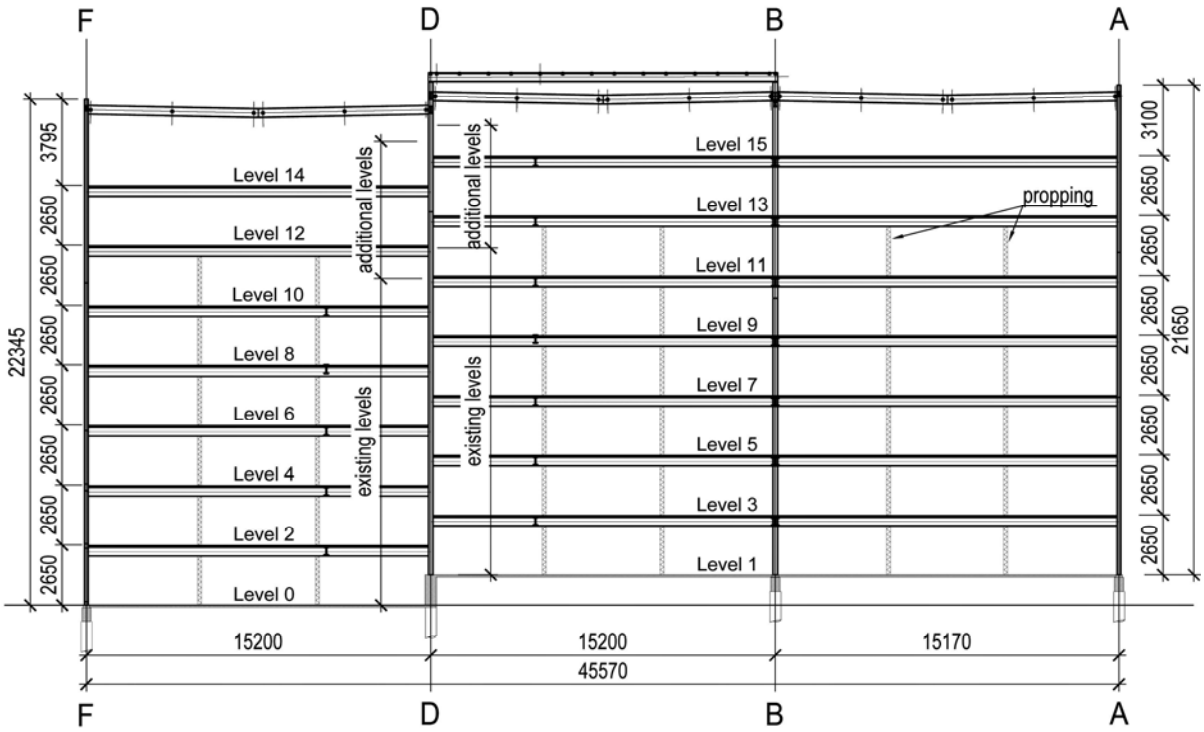

Figure 7. (a) Stud welding, (b) Installed connectors before grouting.

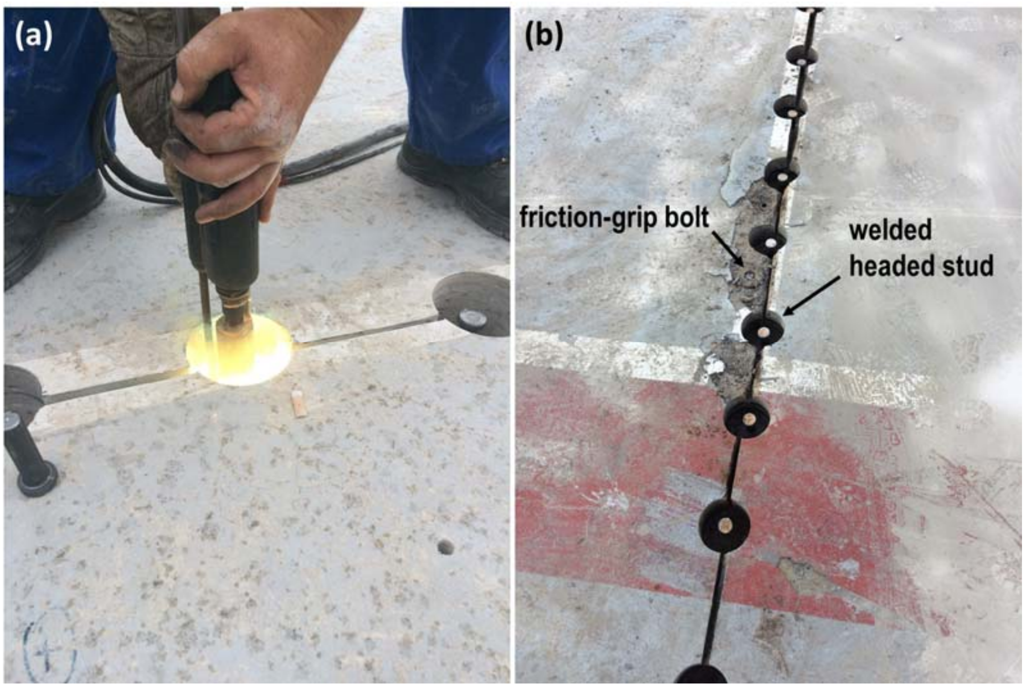

Figure 8. Novel demountable shear connection.

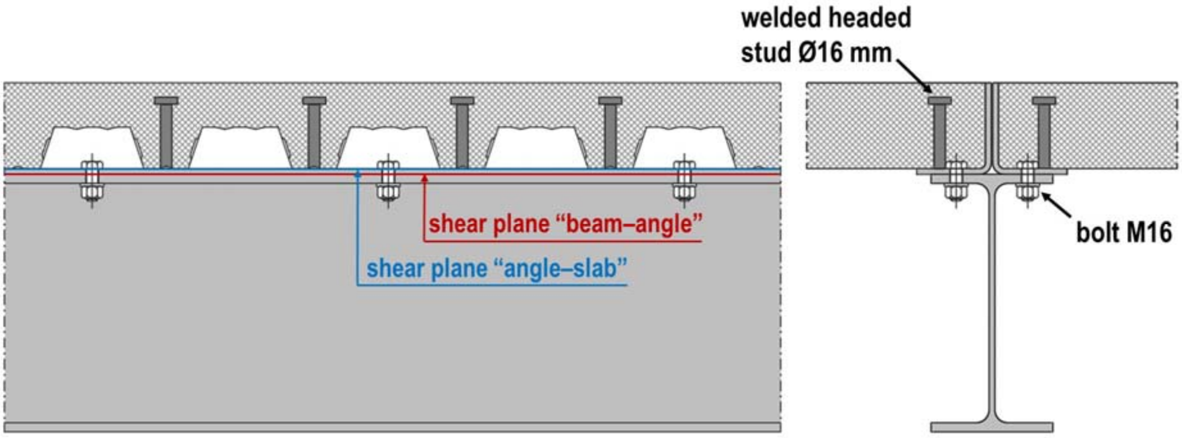

Figure 9. Floor structure demounting: separation of the slab from steel beams.

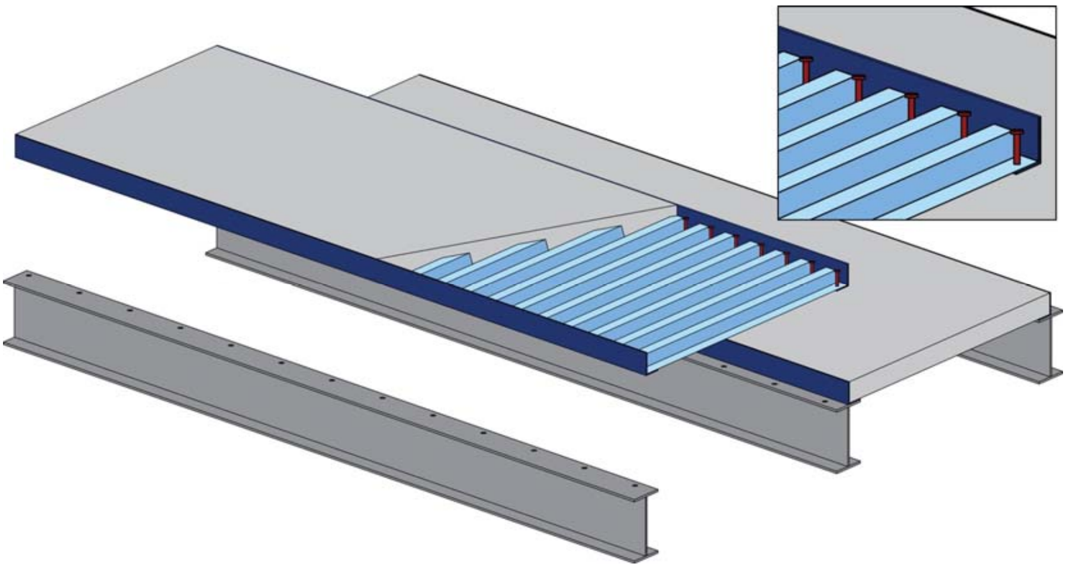

Figure 10. Push-out specimen layout: (a) Series DLU, (b) Series S.

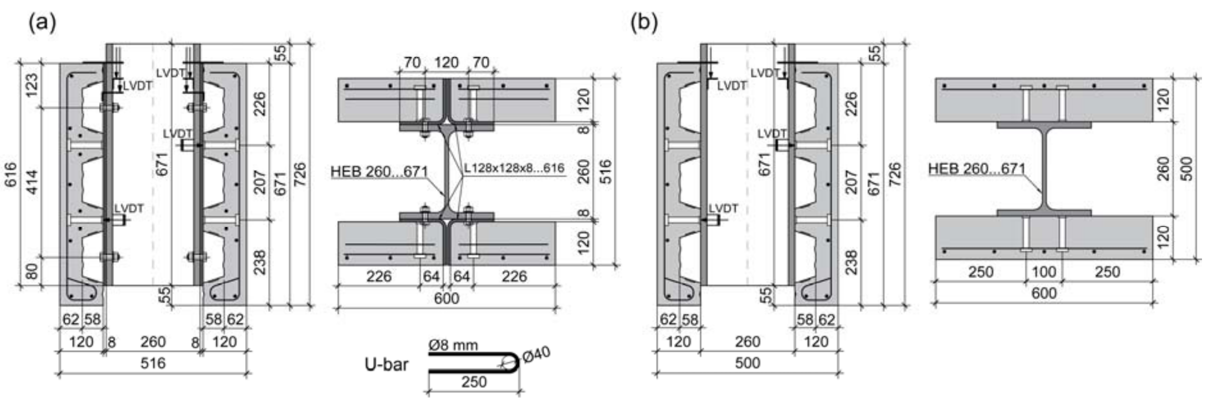

Figure 11. Averaged load–slip curves obtained during experimental testing.

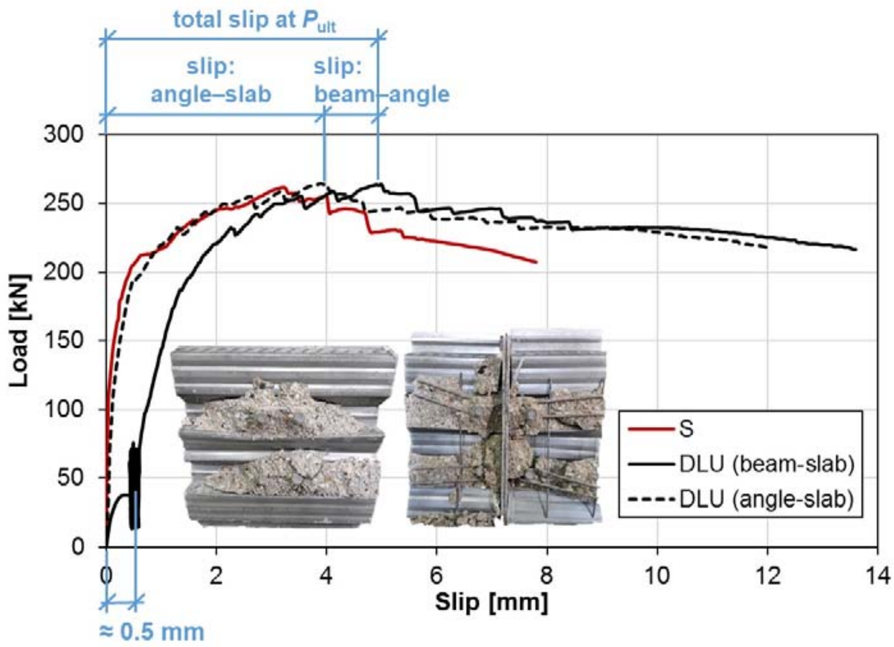

Figure 12. Numerical models of push-out tests: (a) Model DLU, (b) Model S.

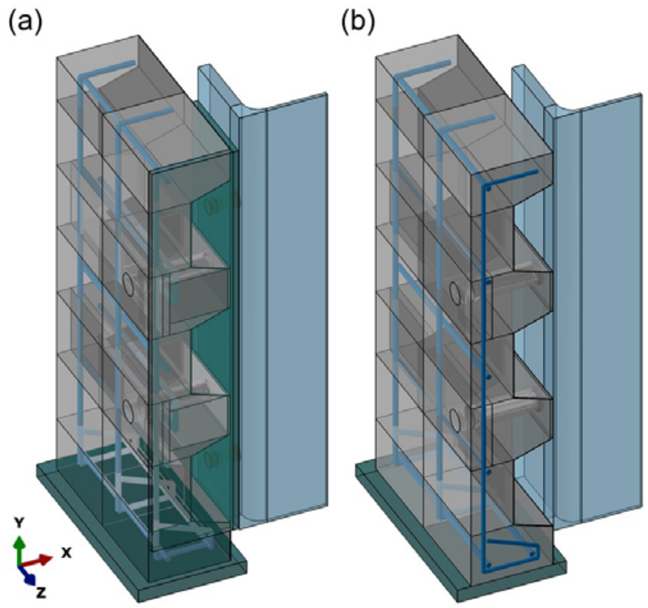

Figure 13. Comparison of experimental and numerical load–slip curves.

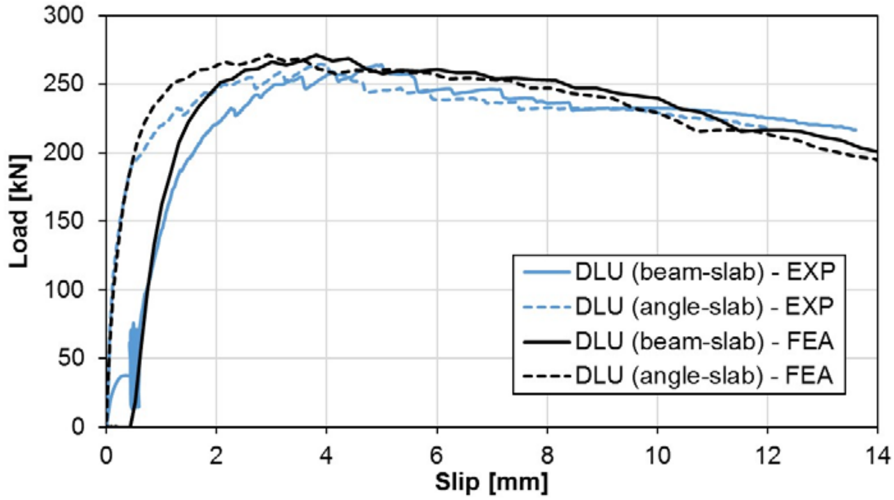

Figure 14. (a) Concrete damage at the slip of 2 mm, (b) Stress distribution at the slip of 6 mm.

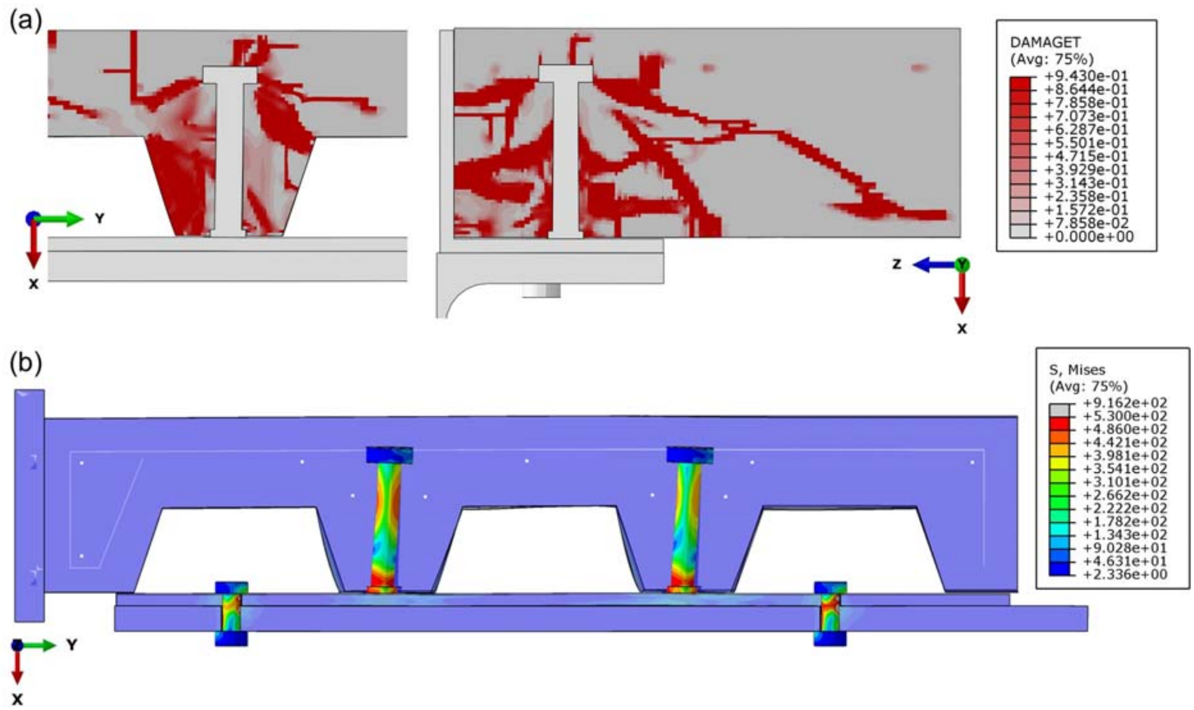

Figure 15. Plastic resistance moments of the composite cross-section when neglecting and when accounting steel angle resistance: (a) Beam L = 15 m, (b) Beam L = 10 m.

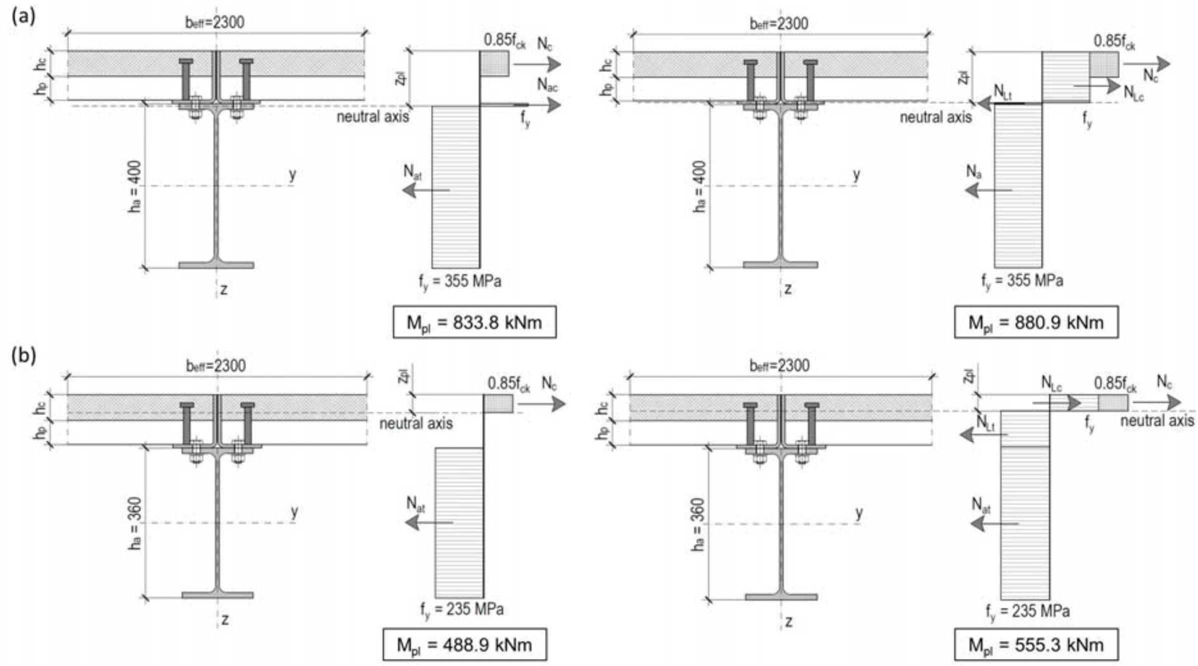

Table 1. Summary of design solutions.

	Original design	Reconstruction design	Proposed solution			Original design	Reconstruction design	Proposed solution		
			Type 1	Type 2	Type 3			Type 1	Type 2	Type 3
Beam span	15 m					10 m				
Beam spacing	2.3 m					2.3 m				
Steel grade	S355					S235				
Beam profile	IPE 400					IPE 360				
Concrete class	C30/37					C30/37				
Concrete slab depth	100 mm		120 mm	120 mm	120 mm	100 mm		120 mm	120 mm	120 mm
Concrete volume per 1 m^2	0.100 m^3		0.085 m^3	0.088 m^3	0.085 m^3	0.100 m^3		0.085 m^3	0.088 m^3	0.085 m^3
Connectors	friction-grip bolts M16 10.9	headed studs Ø22, h_{sc}=90 mm	headed studs Ø16, h_{sc}=100 mm; bolts M16 8.8	headed studs Ø16, h_{sc}=100 mm; bolts M20 8.8	headed studs Ø16, h_{sc}=100 mm; bolts M20 8.8	friction-grip bolts M16 10.9	headed studs Ø22, h_{sc}=90 mm	headed studs Ø16, h_{sc}=100 mm; bolts M16 8.8	headed studs Ø16, h_{sc}=100 mm; bolts M20 8.8	headed studs Ø16, h_{sc}=100 mm; bolts M20 8.8
Spacing between connectors	300 mm	300 mm	headed studs: 207 mm; bolts: 414 mm	headed studs: 300 mm; bolts: 600 mm	headed studs: 332 mm; bolts: 664 mm	300 mm	300 mm	headed studs: 207 mm; bolts: 414 mm	headed studs: 300 mm; bolts: 600 mm	headed studs: 332 mm; bolts: 664 mm
Additional elements	PVC tube, steel plate, helical reinforcement	-	angles; profiled steel sheeting [19]	angles; profiled steel sheeting [20]	angles; profiled steel sheeting [21]	PVC tube, steel plate, helical reinforcement	-	angles; profiled steel sheeting [19]	angles; profiled steel sheeting [20]	angles; profiled steel sheeting [21]
Number of connectors per cross-section	2	1	2	2	2	2	1	2	2	2
Total number of connectors	100	50	146 (headed studs) + 72 (bolts)	100 (headed studs) + 50 (bolts)	90 (headed studs) + 44 (bolts)	66	33	96 (headed studs) + 48 (bolts)	66 (headed studs) + 32 (bolts)	60 (headed studs) + 30 (bolts)

Table 2. Comparison of the headed stud and bolt design shear resistance.

Headed stud	Design shear resistance of the headed stud, P_{Rd} [kN]	Bolt	Design shear resistance of the bolt, F_{Rd} [kN]	Ratio, P_{Rd} / F_{Rd}	Ratio, $2P_{Rd}$ / F_{Rd}
Ø16 mm in profiled sheeting [19]	29.16	M16 8.8	77.21	0.38	0.75
Ø16 mm in profiled sheeting [20,21]	34.74	M20 8.8	120.64	0.29	0.58

Table 3. Experimental push-out test results.

Series	Concrete compressive cube strength	Ultimate load		Stiffness	Max. slip at $0.9P_{ult}$
		Mean	CoV	Mean	Mean
	$f_{c,cube}$ [MPa]	P_{ult} [kN]	[%]	k_{sc} [kN/mm]	δ_u [mm]
DLU	46.6	265.4	1.28	18	9.17
S	43.7	264.6	5.05	75	4.70

Table 4. Comparison of experimental and numerical results.

Series	Ultimate load		Ratio
	Experiment	FEA	
	$P_{ult,exp}$ [kN]	$P_{ult,fea}$ [kN]	$P_{ult,fea}/P_{ult,exp}$
DLU	265.4	271.6	1.02
S	264.6	262.5	0.99

Table 5. Deflections in the beam mid-span.

		Mid-span deflection				
Span	Material grade	Execution stage			Composite stage	
		dead weight	initial slip	imposed load	dead weight	imposed load
[m]		[mm]	[mm]	[mm]	[mm]	[mm]
15	S355	80.8	18.6	23.4	10.1	23.2
10	S235	22.3	13.6	6.6	2.6	6.0